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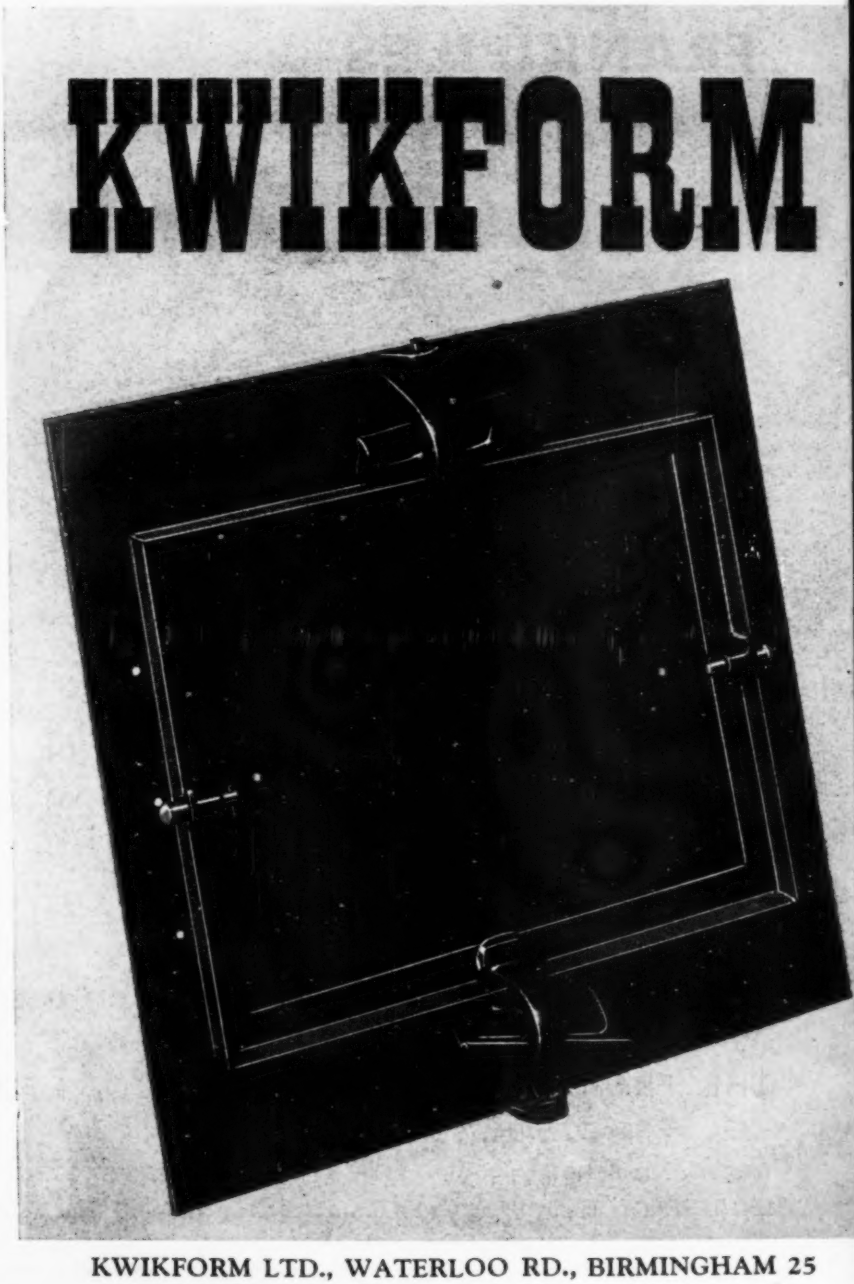
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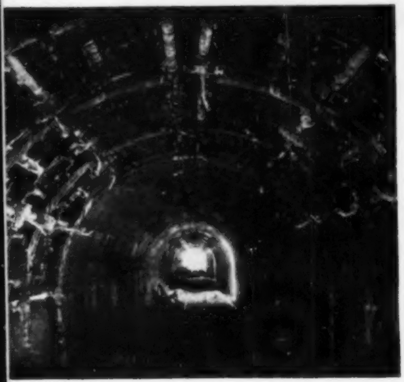

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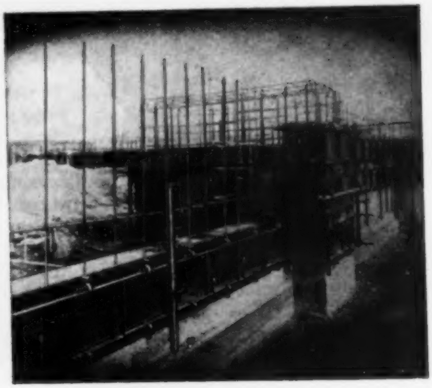

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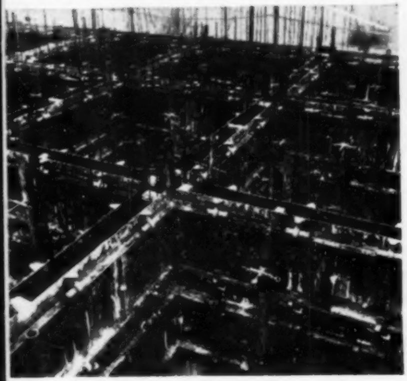

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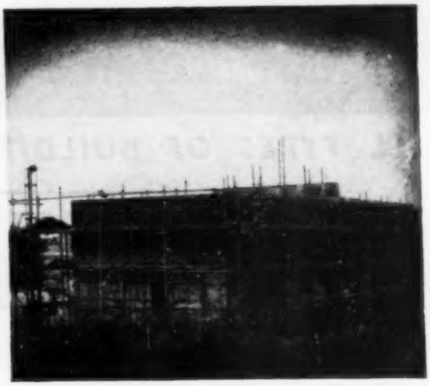

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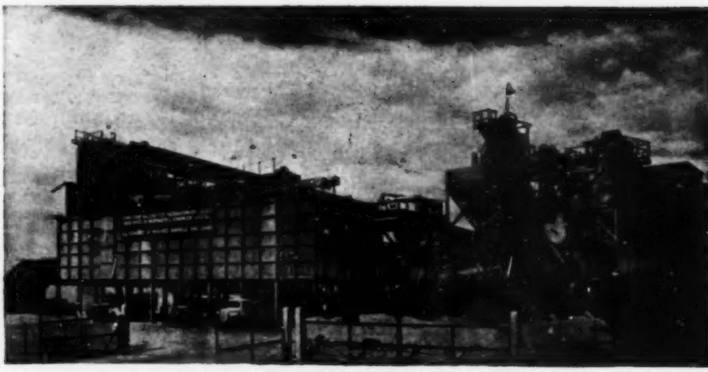

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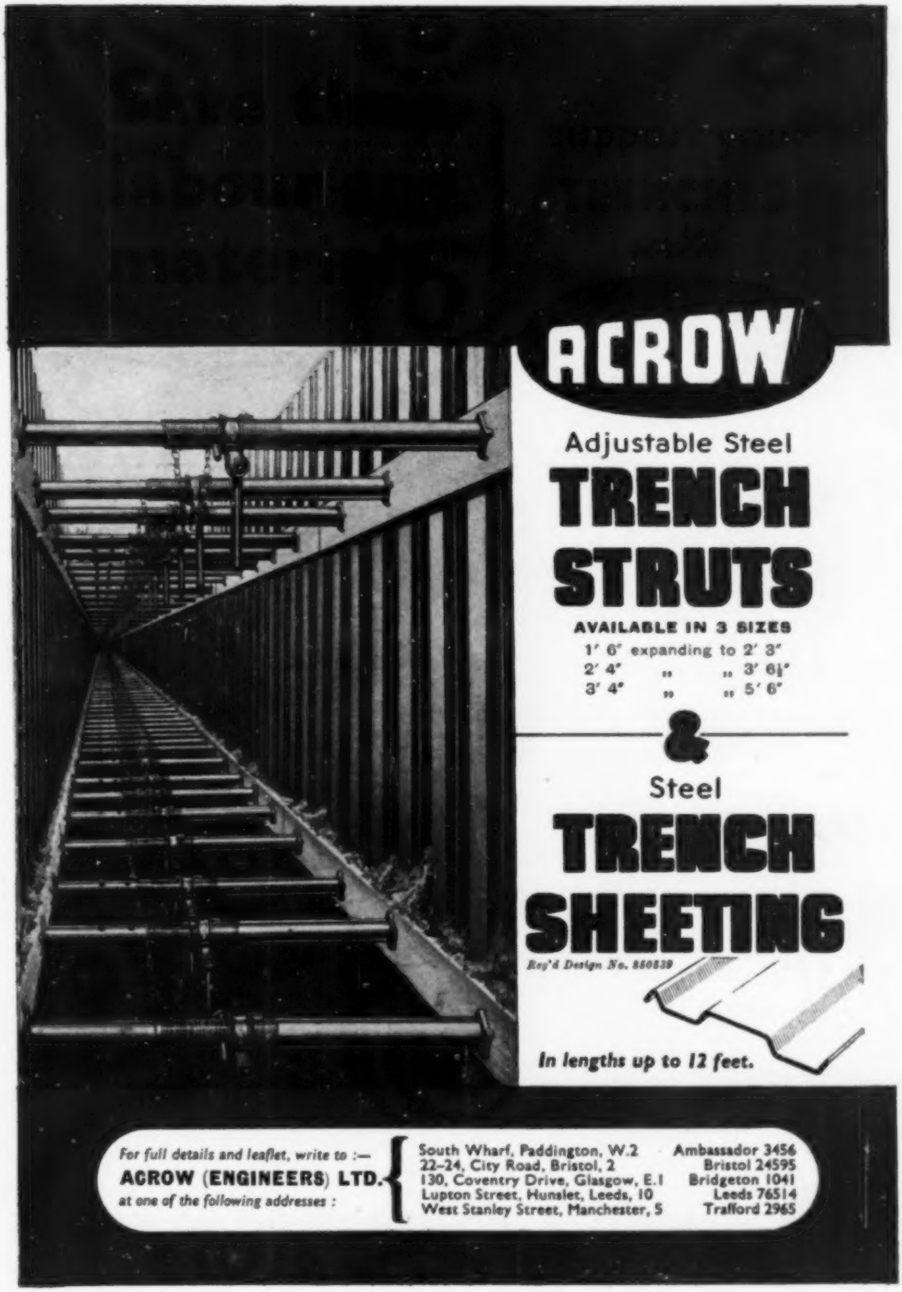
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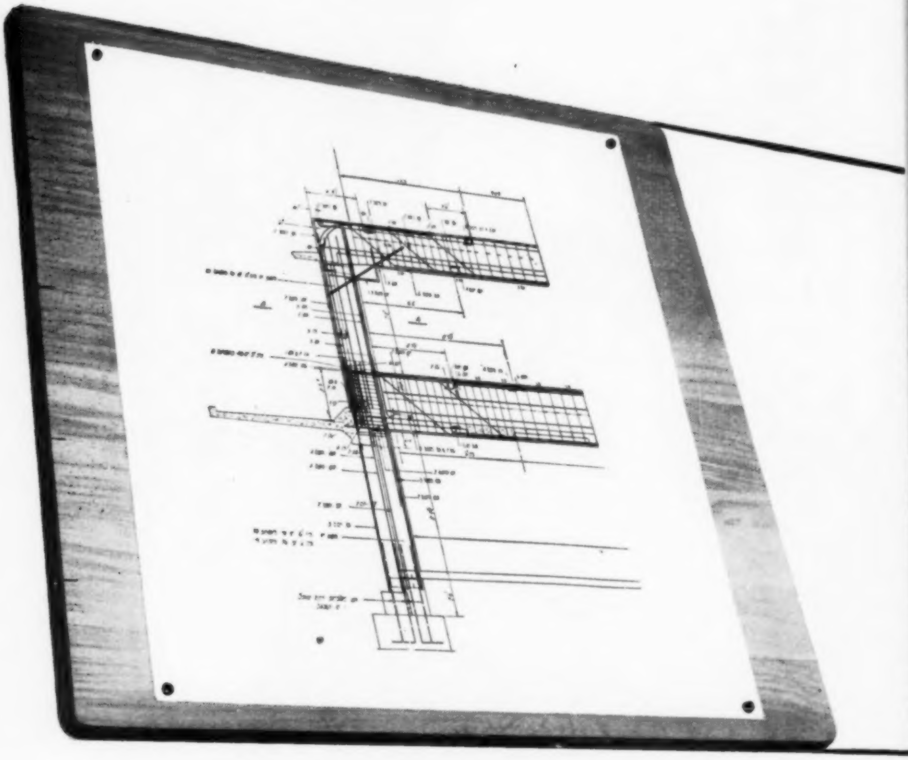

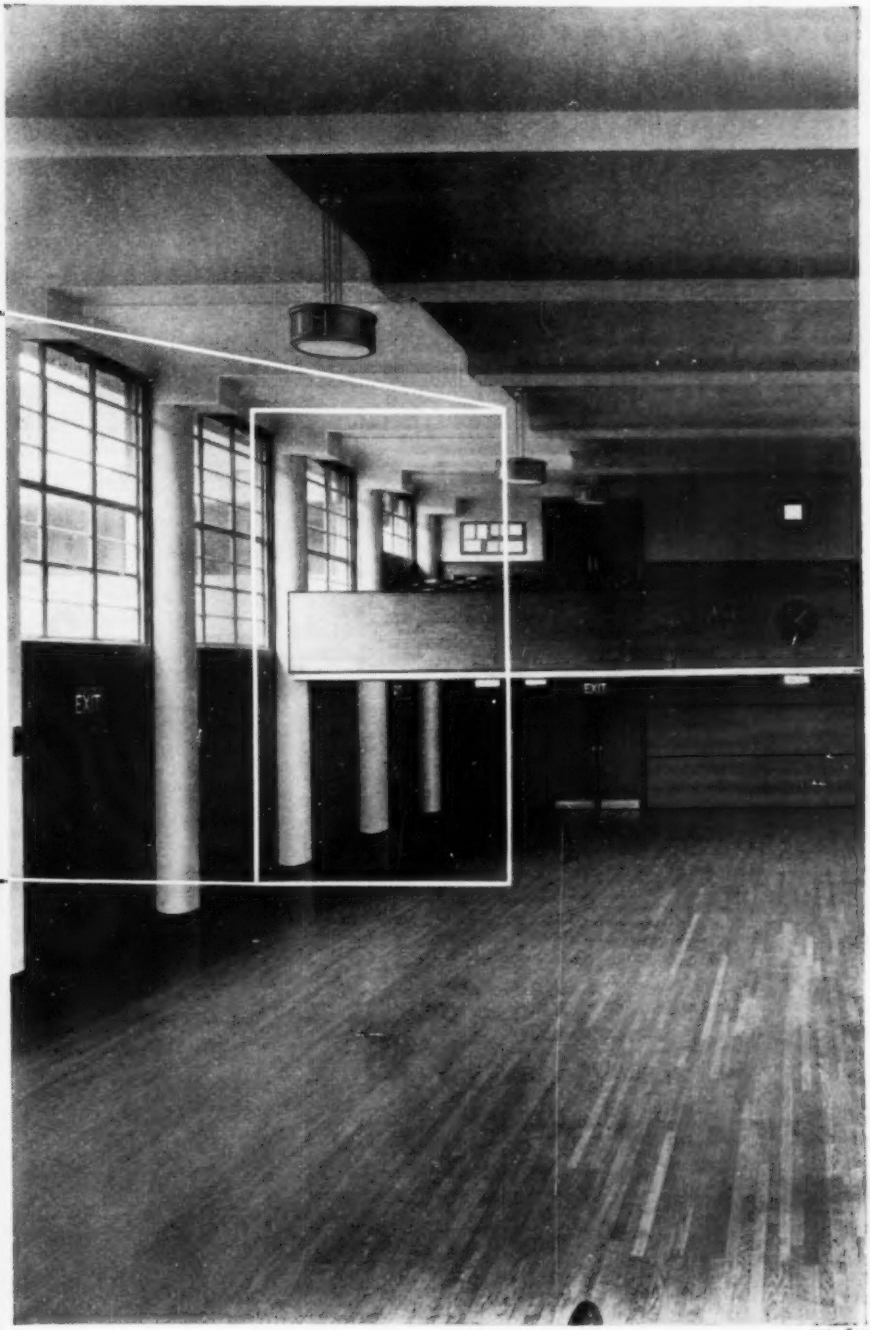

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CONCRETE
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Volume XLVI. No. 10. LONDON, OCTOBER, 1951.

EDITORIAL NOTES

Proposed Metric System in Great Britain.

ONE of the recommendations of the Committee on Weights and Measures Legislation is that the Imperial system of measurement should be abolished in favour of the metric system, the change being made during a period of about twenty years. As is pointed out in the report of the Committee (published by H.M. Stationery Office, price 3*s.* 6*d.*), both systems are legal in Britain at present, although the metric system is not used domestically or extensively in trade other than in the scientific industries. Whereas the metric system is used generally outside the English-speaking countries the Imperial system is used in most English-speaking countries including the United States of America, although some of the standards in the United States differ in magnitude from those bearing the same name in Great Britain. The metric standards in this country differ very slightly from those in use on the Continent, as do also those used in the United States, and the metric standards in Britain and the United States are not identical. There is therefore a lack of harmony between systems that are superficially in agreement, and for this reason the Committee deems it essential for this country, if possible concurrently with the United States of America, to adopt the international metric standards in place of the present Imperial or national metric systems.

In more leisurely days there was less need to simplify methods of measurement, and it was probably not until the present century that the advantages of the metric system were strongly advocated in Great Britain. The British nation was then much more insular than it is to-day, and it was perhaps excusable that a nation of great empire builders and the leaders of industrialisation should expect other nations to adapt their ideas to the British. A view of the last century is expressed by Professor Rankine in one of his rhymes (for Rankine wrote verse as well as works on mechanics). The rhymes are entitled " The Three-foot Rule " and the first two verses are :

> When I was bound apprentice, and learned to use my hands,
> Folk never talked of measures that came from foreign lands :
> Now I'm a British workman, too old to go to school ;
> So whether the chisel or file I hold, I'll stick to my three-foot rule.
>
> Some talk of millimetres, and some of kilogrammes,
> And some of decilitres, to measure beer and drams ;
> But I'm a British workman, too old to go to school,
> So by pounds I'll eat, and by quarts I'll drink, and I'll work by
> my three-foot rule.

Whether Rankine was expressing his own views or those of most of the people of eighty or so years ago we do not know. But aversion to a radical change which will cause much trouble, and particularly if it calls for mental effort and affects everyday life, is a natural reaction which is as evident to-day as it has been throughout history, and we may expect as much opposition to-day as ever before to a proposal to change standards of measurement that have been in use for centuries. English-speaking nations are becoming accustomed to the use of metric measurements in some directions, such as kilowatts in electricity supply, metres in some sports, cubic centimetres in engine capacities, and so on, but these are accepted grudgingly and are meaningless to most people until they have been converted to yards or horsepower, or in the case of kilowatts remain meaningless unless an attempt is made to convert kilowatts to other measurements. It is a waste of time and mental effort to work or think in terms of one system and then to convert the results, and it will often happen that the converted results are unpractical. English-speaking users of the metric system should be so familiar with it that it is as natural to think in this system as in the Imperial system. Engineers are not unfamiliar with the metric system because it is used in technical literature on the continent and in physics and chemistry, and it is common for laboratory tests of materials to be expressed in the metric system. The use of decimals certainly eases calculation, and decimals are invariably used in calculations in the Imperial system in structural engineering.

The use of the metric system in Great Britain has now been advocated for many years, and there is no doubt that when they were accustomed to it the wives and workmen, the shopkeepers and manufacturers, would appreciate its simplicity as much as do scientists and mathematicians. The advantages of the system have been pressed often in the past and ignored by successive governments in easier times, for the dislocation that it would cause has always led to the rejection of the proposed change. Indeed the ardent people who advocated the change were often looked upon as fanatics whose proposal was unpractical. Such a change, which was considered unpractical in the past, may well be thought to be impossible in the foreseeable future. A period when the nation is making armaments against time and when an increase in exports is an immediate and vital necessity is hardly the best time to change our standards of measurement, with the consequent delay and confusion not only while the change is being made but for as long as machines and goods made to the present Imperial standards are in use. In the past such proposals have met with little public approval because of their cost and also because of the desire of any one generation not to cause so much trouble in its own time for the benefit of the next generation. It is likely that this report will be met with the same reservations, and that the difficulties of our times will be sufficient reason for leaving the choice to the next generation, who can pass it on to the succeeding generation if it so desires. The metric system would be a great boon to the country to-day if a previous generation had had the courage to make the change, but it is not likely that the present generation will think it wise to risk disorganisation now for the sake of making life easier for posterity. We may not work so hard or live so well as Rankine's British workman, but British resistance to change is as strong as ever.

The Ultimate Resistance of Prestressed Concrete Beams.

By P. W. ABELES, D.Sc. (Vienna).

THE factor of safety of a beam is the ratio of the ultimate moment of resistance to the moment of resistance at working load. For a prestressed concrete beam, the factor of safety cannot be derived from the resultant stresses at working load because these stresses are composed of the effective prestress (compressive or tensile) and the stresses due to the working load, and are based on a rectilinear distribution in an elastic homogeneous material.

At failure, however, the concrete beam is cracked and in a plastic state and the condition of stress is entirely different. Consequently it is necessary to investigate separately the stresses at working load and at failure. In this article the ultimate resistance to bending is considered, and it is intended to deal with the elastic state in a later article. The terms "under reinforced" and "over reinforced," which are commonly applied to beams in which failure is due primarily to failure of the steel and concrete respectively, are used with these meanings in the following.

The magnitude of the factor of safety required depends on whether the resistance of the steel or concrete is considered. A factor of safety of two should be sufficient for the tensile resistance, but 2·5 to 3 is required for compressive resistance because the strength of the concrete is not known accurately and may vary greatly. It may be better to use different load factors for the dead and live loads, for example, with regard to the steel 1·5 for the dead load and 2·5 for the live load and similarly increased values with regard to the concrete. The same value ($FS = 2$) can be used for the steel and concrete if the compressive strength of the concrete is reduced to the lowest probable value by including a reduction factor of $\frac{2}{2\cdot5}$ $(= 0\cdot8)$ or $\frac{2}{3}$ $(= 0\cdot67)$.

The two common methods of prestressing, which must be considered separately, are (1) Pre-tensioning, that is when the wires are stretched before the concrete is placed and released after the concrete has attained sufficient strength, and are bonded to the concrete, and (2) Post-tensioning, that is when the wires or bars are stretched when the hardened concrete can resist the compression applied by the anchors at each end, the wires not being bonded to the concrete at the time of stretching. Professor R. H. Evans * then was probably the first to describe the difference in behaviour of beams with bonded and non-bonded wires. Beams with pretensioned wires and beams in which post-tensioned wires are efficiently bonded to the concrete by grouting are considered as beams with bonded wires. Beams with post-tensioned wires without grouting or with inefficient grouting are considered as beams with non-bonded wires or bars.

With bonded wires, failure occurs as in an ordinary reinforced concrete beam. The magnitude of the prestress in an under-reinforced beam has little influence on the ultimate load. High-strength steel wire has generally no distinct yield point and, if the percentage of steel is low and the wires are small, a stress

* "Relative Merits of Wire and Bar Reinforcement in Prestressed Concrete Beams." Journal of the Institution of Civil Engineers, February, 1942.

that by calculation is equal to or exceeds the tensile strength will be reached because the greatest extension of the wires occurs only at the cracks, that is only for a fraction of the length of the beam. At failure, the neutral axis rises considerably, due to either a tensile fracture of the wires or excessive elongation of the steel, followed by crushing of the concrete. The minimum load that will cause failure can be determined, and considerable deflection and cracking indicate that failure is imminent.[1] Failure of an over-reinforced beam with bonded wires is due to crushing of the concrete without substantial elongation of the wires, and occurs suddenly when the deflection is relatively small and cracking does not indicate immediate collapse. The magnitude of the load that causes failure is doubtful and depends on the strength and plasticity of the concrete and shape. The crushing strength is about equal to the prism strength, which may be from 60 per cent. to 90 per cent. of the cube strength.

In a beam non-bonded post-tensioned wires or bars act as an independent tie, freely extending between the end anchors, and do not fracture. The calculated stress at failure is consequently never so great as the tensile strength, since extension of the wires exists throughout the length of the beam. Failure of the beam is always a result of the crushing of the concrete as a consequence of the great deflection, the amount of which depends on the initial prestress, the stress-strain relationship of the wire, the strength and plasticity of the concrete, and the shape of the moment diagram. The percentage of steel, which has a particular influence with bonded wires and distinguishes under-reinforced and over-reinforced beams, is of minor importance in a beam with non-bonded wires, and the size of the wire or bar is not restricted, but the elastic limit should be high to avoid excessive extension.

Design Formulæ for Beams with Bonded Wires.

In view of the difference in behaviour of beams with bonded or non-bonded wires, it is impracticable to devise a formula for the design of all beams, but it is possible to adopt the same principal formulæ and use different values of the maximum stress in the wires at failure. In this journal for June, 1946, the writer gave a formula based on the condition at ultimate load for the calculation of the total area of reinforcement (A_t) composed of stretched wires (A_{ts}) and unstretched bonded wires (A_{tu}). The formula can also be stated as

$$A_t = A_{ts} + A_{tu} = \frac{M_w . FS}{a_p t_{max.}} \qquad \text{(1)}$$

in which M_w is the bending moment due to the design load, FS the factor of safety, and $t_{max.}$ the tensile stress in the steel at failure. The lever arm a_p is that of a cracked section assuming a rectangular distribution (plastic) of stress c_p, that is the prism strength. Tests show that this simple approximation[2] is reliable for under-reinforced beams, and that in stretched and unstretched wires up to 0·2 in. diameter, if the bond is efficient, $t_{max.}$ is equal to the tensile strength of the wire ($t_{ult.}$). Otherwise, with bonded unstretched reinforcement, $t_{max.}$ is 85 per cent. to 100 per cent. of $t_{ult.}$, the proportion depending on the efficiency of the bond (which depends on the size and surface of the wire), the stress-strain characteristics of the steel, and the strength of the concrete. For post-tensioned grouted wires $t_{max.}$ may be between 75 per cent. and 100 per cent. of $t_{ult.}$ depending

on the efficiency of the bond due to the grout. If the wire has a definite yield point, $t_{max.}$ is the yield-point stress t_y. Note that $a_p t_{max.} A_t = M_{ult.}$.

Balanced Design with Bonded Wires.

The boundary between under-reinforcement and over-reinforcement, that is the condition of " balanced design ", is not readily established, but it is important to have some idea of the boundary condition because of the different behaviour of the two types of beam. Safe limiting conditions for the balanced design of a rectangular beam can be determined as follows. Let $\bar{A}_t$ be the area and $\bar{p}$ the percentage of reinforcement required for balanced design ; then $\bar{p} = \frac{100\bar{A}_t}{bd}$. It is assumed that the compressive stress in the concrete is equal to c_p (considered to be 0·6 of the cube strength c_u) and is constant from the compressive edge to the neutral axis, the depth n_p to which is assumed not to exceed half the effective depth d. Any prestress effective in the compressive zone at failure must be deducted from $0{\cdot}6c_u$. The greatest resistance $C_{max.}$ which can be developed in the concrete is $0{\cdot}3bdc_u$. For this condition n_p is $0{\cdot}5d$ and $a_{min.}$ is $0{\cdot}75d$, and the moment of resistance $M_{max.}$ for the balanced design is $0{\cdot}75dC_{max.}$ (*Fig.* 1), that is

$$M_{max.} = 0{\cdot}225bd^2c_u \quad . \quad . \quad . \quad . \quad . \quad (2)$$

In a beam with pre-tensioned wires the ultimate resistance $T_{ult.}$ of the steel is $A_t t_{ult.}$, or generally

$$T_{ult.} = (A_{ts} + A_{tu}K_u)t_{ult.} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

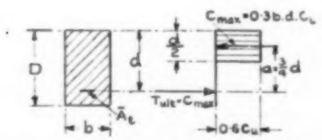

Fig. 1.

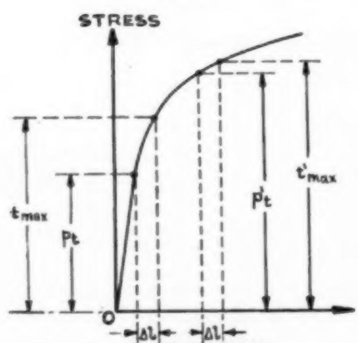

Fig. 3.

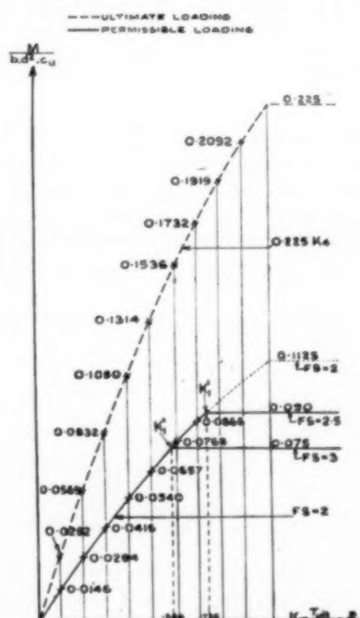

Fig. 2.

in which K_u varies from 0·85 to unity. In a balanced design, $C_{max.} = T_{ult.}$ if the factors of safety for the steel and concrete are equal.

In a beam with efficiently-grouted post-tensioned wires, the ultimate resistance of the wires is generally $(A_{ts}K_g + A_{tu}K_u)t_{ult.}$, in which K_g is from 0·7 to unity depending on the efficiency of the grout. All considerations of bonded conditions relate only to wires, since test data of bars are not available.

Table I gives values of the factors $K_2\left(=\frac{n_p}{d}\right)$, $K_3\left(=\frac{a_p}{d}\right)$, and $K_4\left(=\frac{M_{ult.}}{M_{max.}}\right)$ for various values of $K_1\left(=\frac{T_{ult.}}{C_{max.}}\right)$. If $T_{ult.}$ is less than $C_{max.}$ the beam is under-reinforced, but if greater the beam is over-reinforced and $M_{ult.} = M_{max.}$ for which formula (2) applies. *Fig.* 2 shows the value of $\frac{M}{bd^2c_u}$ plotted against K_1, which can also be considered as the ratio of $p\left(=\frac{100A_t}{bd}\right)$ to $\bar{p}$, which, in accordance with the assumption in *Fig.* 1 for balanced design, is

$$\bar{p} = \frac{100\bar{A}_t}{bd} = \frac{30c_u}{t_{max.}} \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Curves are given for the ultimate and permissible loads. Thus the permissible bending moment for safe balanced design is $0{\cdot}09bd^2c_u$ and $0{\cdot}075bd^2c_u$ if FS is 2·5 and 3 respectively, the corresponding values of K_1 or $\frac{p}{\bar{p}}$ being 0·735 and 0·586 as indicated in *Fig.* 2. These values are obtained from $K_4 = \frac{4K_1K_3}{3}$, on which *Table* I is based. Since $K_2 = \frac{K_1}{2}$, $K_3 = 1 - \frac{K_1}{4}$. Therefore $K_4 = \frac{4K_1}{3}\left(1 - \frac{K_1}{4}\right)$.

TABLE I.—COEFFICIENTS FOR ULTIMATE MOMENT OF RESISTANCE OF RECTANGULAR BEAMS.

$K_1 = \frac{T_{ult.}}{C_{max.}}$	$K_2 = \frac{n_p}{d}$	$K_3 = \frac{a_p}{d}$	$K_4 = \frac{M_{ult.}}{M_{max.}}$	Notes	
0·1	0·05	0·975	0·130	—	Under-reinforced beams
0·2	0·10	0·95	0·253	—	
0·3	0·15	0·925	0·370	—	
0·4	0·20	0·90	0·480	—	
0·5	0·25	0·875	0·584	—	
0·6	0·30	0·85	0·680	—	
0·667	0·333	0·833	0·740	Limit : $FS_{concrete} = 3$	
0·7	0·35	0·825	0·770	—	
0·8	0·40	0·80	0·853	Limit : $FS_{concrete} = 2{\cdot}5$	
0·9	0·45	0·775	0·930	—	
1·0	0·50	0·75	1·00	Balanced design : $FS_{concrete} = 2$	

The percentage of steel for balanced design is reduced to

$$0{\cdot}735\bar{p} = \frac{22{\cdot}05c_u}{t_{max.}}$$

for $FS = 2{\cdot}5$, and to $0{\cdot}586\bar{p} = \dfrac{17{\cdot}58c_u}{t_{max.}}$ for $FS = 3$. If $c_u = 7000$ lb. per square inch, and $t_{max.} = t_{ult.} = 224{,}000$ lb. per square inch, from (4) $\bar{p}$ is 0·94 per cent. for actual balanced design, but is reduced to 0·69 and 0·55 per cent. for safe permissible balanced designs with FS equal to 2·5 and 3 respectively.

In the foregoing the magnitude of the initial prestress is disregarded, which is justified for a beam for which K_1 is small (even if the prestress is very small), or if the value of K_1 approaches unity provided that the initial prestress is from $0{\cdot}6t_{ult.}$ to $0{\cdot}7t_{ult.}$. There is a possibility that, for values of K_1 approaching unity, lower values of $M_{max.}$ apply if the prestress is below $0{\cdot}6t_{ult.}$. At present, although

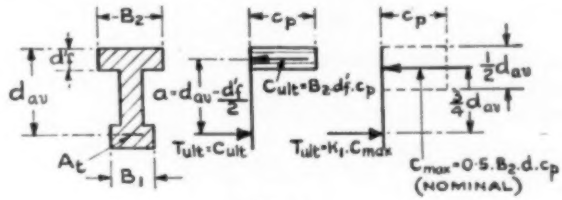

Fig. 4.

there are no data available regarding this matter, the possibility of such a reduction of the maximum resistance if there is a small prestress should not be overlooked.

I-Beams with Bonded Wires.

It appears to be permissible in an under-reinforced I-beam (*Fig.* 4) to assume the stress in a shallow top flange to be greater than $0{\cdot}6c_u$, depending on the actual prism strength, even up to, say, $0{\cdot}9c_u$. It may be justifiable to use a greater stress than $0{\cdot}6c_u$, say, $0{\cdot}75c_u$, if exact data are not available. *Table* I can be applied to I-beams by calculating a nominal maximum value of $M_{max.}$ from $0{\cdot}375B_2d_{av.}^2c_p$, while the actual maximum value for depth d_f' and lever arm $d_{av.} - \dfrac{d_f'}{2}$ is given by

$$M_{max.} = B_2d_f'\left(d_{av.} - \frac{d_f'}{2}\right)c_p.$$

If $c_p = 0{\cdot}75c_u$, the nominal value is $0{\cdot}28B_2d_{av.}^2c_u$.

Beams with Non-bonded Wires or Bars.

Since the greatest stress in the steel at failure is governed by the greatest extension, it mainly depends on the stress p_t in the steel at the time when the force is applied to the concrete. In beams with post-tensioned wires or bars, the force is transmitted from the wires or bars to the concrete by the anchors,

and not by bond as is the case with pre-tensioned wires or bars. If p_t is $0{\cdot}65t_{ult.}$ a safe value for $t_{max.}$ is $0{\cdot}75t_{ult.}$, and $T_{ult.}$ is then $0{\cdot}75A_t t_{ult.}$. If p_i is less than $0{\cdot}65\,t_{ult.}$ smaller values of $t_{max.}$ should be assumed. The governing factor is the maximum elongation Δl possible, and $t_{max.} - p_i$ should in each case correspond to the same strain (*Fig.* 3). However, a simple and safe approximation is that $t_{max.} = p_t + 0{\cdot}1t_{ult.}$, which gives a more unfavourable condition for low values of p_t but shows that it is not advisable for the initial prestress p_i to be too small. Generally, therefore, the tensile resistance is given by

$$T_{ult.} = A_{ts}(p_t + 0{\cdot}1t_{ult.}) + K_u A_{ts} t_{ult.} \qquad \qquad (5)$$

It is seen that unstretched well-bonded wire is much more efficient as regards the ultimate stress than non-bonded wires or bars with a small prestress. *Table* I can be used for the design of beams with non-bonded wires, but the reduced value of $T_{ult.}$ must be used.

Examples.

Rectangular Beams.—Assume $d = 12$ in., $b = 6$ in., $c_u = 9000$ lb. per square inch, and $t_{ult.} = 100$ tons per square inch. All wires 0·2 in. diameter. Then $c_p = 0{\cdot}6c_u = 5400$ lb. per square inch.

$$C_{max.} = 6 \times 6 \times 5400 = 194{,}000 \text{ lb.} = 86{\cdot}8 \text{ tons.}$$
$$M_{max.} = 0{\cdot}225 \times 6 \times 12^2 \times 9000 = 1{,}750{,}000 \text{ in.-lb.}$$

(*a*).—Provide ten wires pre-tensioned ($A_t = 0{\cdot}314$ sq. in.).

$$T_{ult.} = 31{\cdot}4 \text{ tons.} \quad K_1 = \frac{T_{ult.}}{C_{max.}} = 0{\cdot}362. \quad \text{From } Table \text{ I,}$$

$$M_{ult.} = T_{ult.}(K_3 d) = 31{\cdot}4 \times 2240 \times 0{\cdot}91 \times 12 = 768{,}000 \text{ in.-lb.,}$$
$$\text{or } M_{ult.} = K_4 M_{max.} = 0{\cdot}438 \times 1{,}750{,}000 = 766{,}500 \text{ in.-lb. ;}$$

$$\text{or directly, } a = 12 - \frac{31{\cdot}4 \times 2240}{2 \times 6 \times 5400} = 10{\cdot}91 \text{ in., and}$$

$$M_{ult.} = 31{\cdot}4 \times 2240 \times 10{\cdot}91 = 767{,}400 \text{ in.-lb.}$$

(*b*).—Provide twelve wires pre-tensioned ($A_{ts} = 0{\cdot}377$ sq. in.), and eight wires not tensioned ($A_{tu} = 0{\cdot}251$ sq. in.). Assume $K_u = 0{\cdot}95$.

$$T_{ult.} = (0{\cdot}377 \times 100) + (0{\cdot}251 \times 100 \times 0{\cdot}95) = 61{\cdot}5 \text{ tons.}$$

$$K_1 = \frac{61{\cdot}5}{86{\cdot}8} = 0{\cdot}708\,; \quad M_{ult.} = 0{\cdot}777 \times 1{,}750{,}000 = 1{,}360{,}000 \text{ in.-lb.}$$

(*c*).—Provide 48 wires pre-tensioned ($A_t = 1{\cdot}507$ sq. in.).

$T_{ult.} = 1{\cdot}507 \times 100 = 150{\cdot}7$ tons, which is greater than $C_{max.}$. Therefore the beam is over-reinforced and $M_{ult.} = M_{max.} = 1{,}750{,}000$ in.-lb.

(*d*).—Provide 24 wires post-tensioned and not bonded ($A_t = 0{\cdot}754$ sq. in.). Assume $p_t = 68$ tons per square inch and $K_g = 0{\cdot}75$.

$$T_{ult.} = 0{\cdot}754 \times 0{\cdot}75 \times 100 = 56{\cdot}5 \text{ tons.}$$

$$K_1 = \frac{56{\cdot}5}{86{\cdot}8} = 0{\cdot}651.$$

$$M_{ult.} = 0{\cdot}727 \times 1{,}750{,}000 = 1{,}272{,}000 \text{ in.-lb.}$$

(e).—As (d) but $p_t = 40$ tons per square inch. $t_{max.} = 40 + (0{\cdot}1 \times 100) = 50$ tons per square inch.

$$T_{ult.} = 0{\cdot}754 \times 50 = 37{\cdot}7 \text{ tons.}$$

$$K_1 = \frac{37{\cdot}7}{86{\cdot}8} = 0{\cdot}435.$$

$$M_{ult.} = 0{\cdot}516 \times 1{,}750{,}000 = 903{,}000 \text{ in.-lb.}$$

I-BEAM (*Fig.* 4).—Assume that $d = d_{av.} = 12$ in., $B_2 = 8$ in., $d'_f = 4$ in., $b = 2$ in., $c_u = 8000$ lb. per square inch, $c_p = 6000$ lb. per square inch,

$$t_{max.} = t_{ult.} = 120 \text{ tons per square inch,}$$

$K_u = 1$ for unstretched wires because good bond is assured, and $FS = 2$ for steel and 2·5 for concrete. Determine the cross-sectional area of wires required.

(a) To resist the maximum permissible bending moment.

$$C_{ult.} = B_2 d'_f c_p = 8 \times 4 \times 6000 = 192{,}000 \text{ lb.}\,; \quad a_{min.} = d - \frac{d'_f}{2} = 10 \text{ in.}$$

Permissible bending moment M_w due to the working load

$$= \frac{C_{max.} a_{min.}}{(FS_{concrete})} = \frac{192{,}000 \times 10}{2{\cdot}5} = 768{,}000 \text{ in.-lb.}$$

$$A_t = A_{ts} + A_{tu} = \frac{(FS_{steel})M_w}{a_{min.} t_{max.}} = \frac{2 \times 768{,}000}{10 \times 120 \times 2240} = 0{\cdot}57 \text{ sq. in.}$$

(b) To resist bending moment M_w of 500,000 in.-lb.

A nominal $M_{max.}$ for a rectangle must be computed. Since $c_p = 0{\cdot}6 c_u = 6000$ lb. per square inch, the nominal value of c_u is 10,000 lb. per square inch, and $M_{max.} = 0{\cdot}225 \times 8 \times 12^2 \times 10{,}000 = 2{,}590{,}000$ in.-lb.

$$K_1 = \frac{T_{ult.}}{C_{max.}} = \frac{(FS_{steel})M_w}{M_{max.}} = \frac{2 \times 500{,}000}{2{,}590{,}000} = 0{\cdot}386\,;$$ the corresponding value of K_3, from *Table* I, is 0·903. Therefore

$$A_t = A_{ts} + A_{tu} = \frac{(FS_{steel})M_w}{K_3 d t_{max.}} = \frac{2 \times 500{,}000}{0{\cdot}903 \times 12 \times 120 \times 2240} = 0{\cdot}333 \text{ sq. in.}$$

Tests.

Tests made by the writer in association with Mr. C. H. Hockley at the Brixton School of Building show that these formulæ give safe results. In addition to

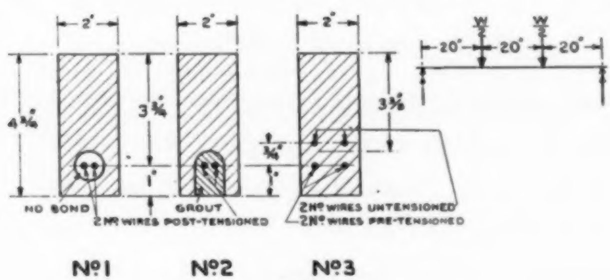

Fig. 5.

TABLE II.—COMPARISON OF CALCULATED RESULTS WITH TESTS.

Beam No.		1	2	3	
No. of stretched wires	Total	2	2	2	
	Bonded	—	2	2	
	Unbonded	2	—	—	
No. of unstretched wires		—	—	2	
$A_t = A_{ts} + A_{tu}$		0·0168	0·0168	0·0336	sq. in.
$d_{av.}$		3·75	3·75	3·375	in.
Initial stretching force P_i		1900	1900	2100	lb.
Initial tensile stress p_i		113,000	113,000	125,000	lb. per square inch
Cube strength c_u of concrete at time of test		6800	6800	8040	lb. per square inch
Failure load $W_{ult.}$		0·525	0·95	1·40	ton
Maximum deflection		0·8	1·3	1·0	in.
Number of cracks in beam		1	5	8	
$M_{ult.} = 10W_{ult.} + M_s$*		5·42	9·67	14·17	in.-tons ($M_s = 0·17$)
$M_{ult.}$	By test	12,150	21,650	31,750	in.-lb.
	By calculation	8,660	17,850	29,500	in.-lb.
$\frac{M_{ult.} \text{ by test}}{M_{ult.} \text{ by calculation}}$		1·40	1·21	1·075	

* M_s is the bending moment due to the weight of the beam.

some beams tested in 1949 and 1950,* three beams (*Fig.* 5) were tested in 1951. The beams were prestressed by No. 12 gauge wires (0·0084 sq. in.) having a tensile strength of 138 tons per square inch. (The stress in some test pieces of wire which broke at the grips was less.) Comparison of the calculated and actual ultimate moments of resistance are given in *Table* II by permission of Mr. D. A. G. Reid, the Principal of the School. The calculations are as follows.

BEAM NO. 1.—$C_{max.} = 0·3 \times 3·75 \times 2 \times 6800 = 15,300$ lb. $M_{max.} = 0·225 \times 2 \times 3·75^2 \times 6800 = 43,100$ in.-lb. $p_t = \frac{1900}{0·0168} = 113,000$ lb. per square inch. An arbitrary safe value of $t_{max.}$ is $p_t + 0·1t_{ult.} = 113,000 + (0·1 \times 310,000) = 144,000$ lb. per square inch. $T_{ult.} = 0·0168 \times 144,000 = 2420$ lb. $K_1 = \frac{T_{ult.}}{C_{max.}} = \frac{2420}{15,300} = 0·158$; from *Table* I, $K_4 = 0·201$; hence $M_{ult.} = 0·201 \times 43,100 = 8660$ in.-lb.

BEAM NO. 2.—$C_{max.}$ and $M_{max.}$ are as for beam No. 1; $t_{max.} = 310,000$ lb.

* "Small Scale Demonstration Tests on Prestressed Concrete Beams." By P. W. Abeles and C. H. Hockley. "The Structural Engineer", June, 1950.

per square inch, and $T_{ult.} = 0{\cdot}0168 \times 310{,}000 = 5210$ lb., $K_1 = \dfrac{5210}{15{,}300} = 0{\cdot}34$, and $K_4 = 0{\cdot}414$. Hence $M_{ult.} = 0{\cdot}414 \times 43{,}100 = 17{,}850$ in.-lb.

BEAM No. 3.—$C_{max.} = 0{\cdot}3 \times 3{\cdot}375 \times 2 \times 8040 = 16{,}250$ lb. $M_{max.} = 0{\cdot}225 \times 3{\cdot}375^2 \times 2 \times 8040 = 41{,}100$ in.-lb. $T_{ult.} = 2 \times 5210 = 10{,}420$ lb., which is twice the value for beam No. 2 corresponding to $K_u = 1$. $K_1 = \dfrac{10{,}420}{16{,}250} = 0{\cdot}641$; $K_4 = 0{\cdot}717$. Hence $M_{ult.} = 0{\cdot}717 \times 41{,}100 = 29{,}500$ in.-lb.

This article is in general agreement with the "First Report on Prestressed Concrete" published by the Institution of Structural Engineers, September, 1951.

Notes.

(1)—There are two exceptions when failure of an under-reinforced beam may occur suddenly, namely, when the precompression is so great that cracking and failure occur simultaneously, that is the beam is over-prestressed, and when the percentage of steel is so small that after cracking of the concrete the wires are incapable of resisting the tensile force. Consequently the percentage should not be less than 0·15 (related to the width at the bottom) if the wire has a tensile strength of 100 tons per square inch, or $\dfrac{15}{t_{ult.}}$ if the strength of the wire is $t_{ult.}$ tons per square inch.

(2)—There are several methods of design such as those, based on the ultimate strain of the concrete in compression and of the steel in tension, of Professor A. L. L. Baker (Journal of Institution of Civil Engineers, February, 1951) and Mr. J. W. H. King (this journal, September, 1950). The latter method is based on the average elongation measured over one or more cracks. If the steel is bonded to the concrete the average strain does not represent the maximum strain on which failure depends and which occurs in the cracks. In the writer's view it is possible to obtain a relationship between the average and maximum steel strains only for beams of the same cross section and made with the same concrete, and having wires of the same quality, size, and strength. Extensive research would be required to obtain reliable design data. Consequently, only approximations are possible at present, and the simplest solution seems to be preferable.

Proposed Concrete Dam in Ceylon.

TENDERS are invited for hydro-electric works at Watawala, Ceylon, including the construction of a mass concrete dam 730 ft. long and 130 ft. high. Further information may be had from the Crown Agents for the Colonies, 4 Millbank, London, S.W.1, the reference number CRE(IB)71489/51 should be quoted.

Plastic Hinges.

PROFESSOR A. L. L. BAKER (Professor of Concrete Technology at the Imperial College of Science and Technology) writes:

I have read the Editorial Note of your September number with great interest, and would like to contribute a few comments.

Recent research has shown that in beams the rotation of plastic hinges is mainly due to yield of the reinforcement, which is accompanied by a rise of the neutral axis. The rotation afforded by the plasticity of the concrete is small in comparison. Provided that the neutral axis rises sufficiently before the concrete in the compressive zone fails by crushing, adequate rotation can be provided and the difficulties of calculation that you suggest exist in connection with tee-beams thus disappear.

Columns and members of frames subjected to compression only, particularly those in which the eccentricity of the direct force is small, present greater difficulty because rotation of plastic hinges which may form depends on the difference of strain which can develop in the concrete across the section. Tests are now proceeding to try to find ways of adjusting reinforcement and binding locally which will ensure that sufficient difference of strain can develop before failure takes place.

Reinforced concrete frames, it is anticipated, may therefore have an advantage over structural steel in the application of the plastic theory. The positions of plastic hinges will be assumed at suitable points and reinforcement adjusted locally until the elastic flexural stiffness of the members and the available plastic rotation of the hinges are within the required limits. Also it is not difficult to calculate approximately by moment distribution, applied to the results obtained by the plastic theory, the stresses under working load to ensure that cracking will not occur.

There should be no narrow or rigid adherence to either the elastic or the plastic theory. Each should be used with intelligent discrimination when it is appropriate. Further research, it is anticipated, will extend the useful application of the plastic theory by simplifying calculations and more precisely evaluating factors of safety.

Book Review.

"Rustungsbau." By H. Kirchner and Adolf Müllenhof Lübeck. 2 vols. (Berlin: Wilhelm Ernst & Sohn. 1951. Prices: Vol. I, 21.50 D.M.; Vol. II, 16.50 D.M.)

VOLUME I deals with the principles of calculation and design of staging for bridges. Data on the strength of materials and permissible stresses are given, and constructional details are treated in great detail. Stagings for use where clearances are required to maintain traffic on railways and waterways are discussed in detail with many examples. Staging for fixed steel-girder bridges with horizontal and curved booms, for arch and suspension bridges and for the movement of bridges are dealt with. The use of boats for floating bridges into position is referred to, and methods of lifting bridges are discussed.

Volume II describes centering for concrete arches, separate sections dealing with the use of struts and braces, fan-shaped arrangements of timbering, staging supported on posts, trestles, and timber piles, and combinations of staging and trusses. Methods of erection and removal in each case are considered, and useful information is given on auxiliary staging required for the transport of building materials and the movement of cranes.

Clear illustrations with details and sizes of the members are a feature of the book.

Publication Received.

"First Report on Prestressed Concrete." The Institution of Structural Engineers. 32 pages. 1951. Price 3s. 6d.

Airfield near Bedford.

AN airfield is to be built at the National Aeronautical Establishment near Bedford at a cost of more than £3,000,000. The contract has been awarded to Messrs. John Laing & Son, Ltd.

Construction of a Large Jetty on Southampton Water.

PILE HAMMERS OF 4 TONS AND 8½ TONS.

THE construction of the jetty for the new refinery for the Esso Petroleum Co., Ltd., at Fawley on Southampton Water, required the use of unusual equipment, some of which was floating. The design of the jetty was described in this journal for September, 1951. Briefly, the main structures comprise a pierhead trestle 2470 ft. long in continuation of which there are two mooring dolphins at each end connected to the pierhead trestle by catwalks. The distance between centres of the outer mooring dolphins is 3200 ft. On the seaward side of the pierhead trestle are four berthing islands, and on the landward side a trestle connecting to a pump-house area and an approach 1900 ft. long. The pierhead trestle, berthing islands, and connecting trestle are built with 32 in. cylindrical hollow precast reinforced concrete piles, reinforced concrete caissons of 36 ft. and 40 ft. diameter, and precast frames. The approach is carried on 16-in. by 14-in. precast piles. The extensive temporary works included a casting yard and an embankment and trestle carrying a railway from the yard to works off the land, where the caissons were constructed.

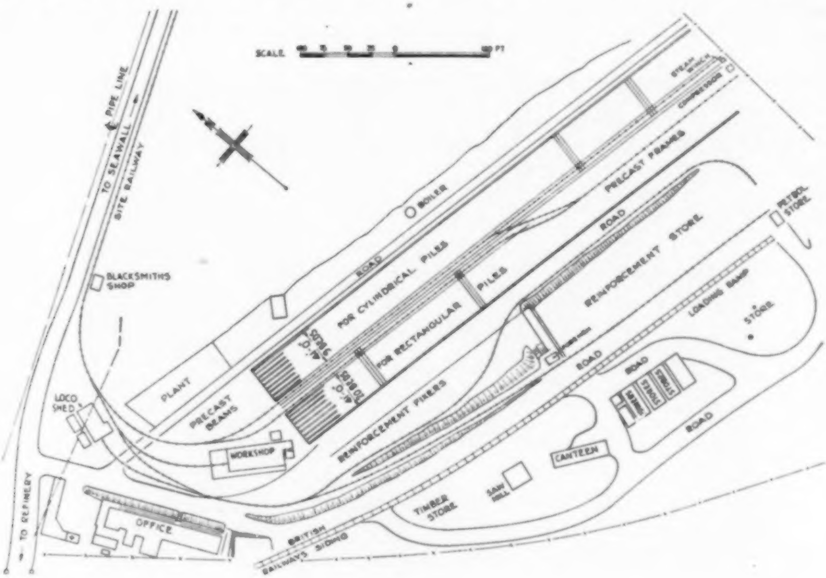

Fig. 1.—Plan of Casting Yard.

Casting Yard.

The casting yard (*Fig.* 1) was on a levelled site about 1000 yd. from high-water line and adjacent to a road and railway, a standard-gauge siding from which ran into the yard. Timber and other stores were unloaded on one side of the siding and reinforcement on the other side where there was a siding from the site railway. There were four principal casting areas. One of these was 41 ft. wide and had nine parallel beds about 600 ft. long for the cylindrical piles. An area of the same width had twenty parallel beds about 230 ft. long for the rectangular piles, and another about

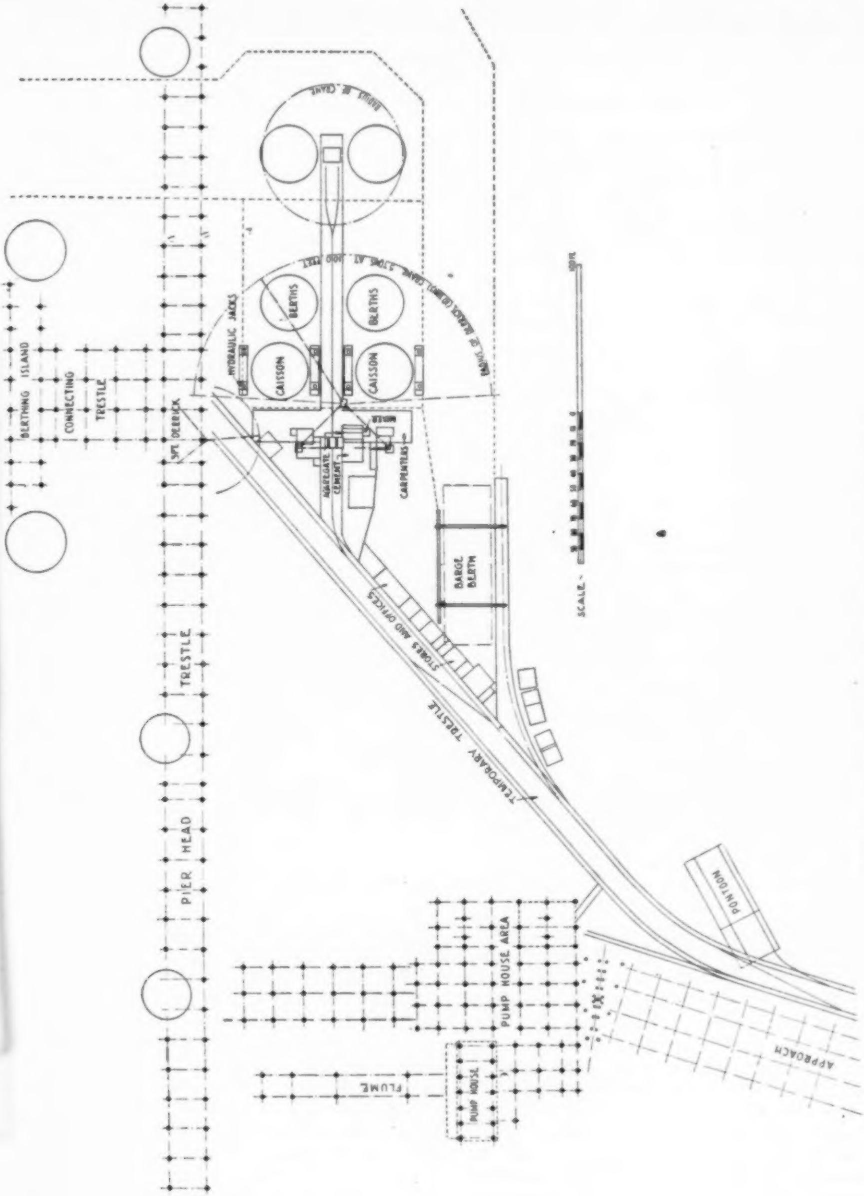

Fig. 2.—Arrangement of Temporary Works.

350 ft. long was used for casting the large frames. A smaller area was used for casting struts, beams, and gangways. At one part of the yard, small concrete blocks to be used as distance-pieces for spacing reinforcement were cast, two men being employed solely on this work.

The paving of the casting yard was generally a 6-in. concrete slab. Each of the two lines of main beds was spanned by two 48-ft. travelling gantries carrying 10-tons chain-blocks. There were also two 4-tons mobile cranes. A boiler supplied steam for heating the castings in winter, and a 115-cu. ft. compressor supplied air for cleaning out the moulds. Power-driven machines were used for bending reinforcement, and equipment was provided for butt welding the long bars in the piles. The quantity of concrete cast daily when the yard was fully in operation was about 50 cu. yd.

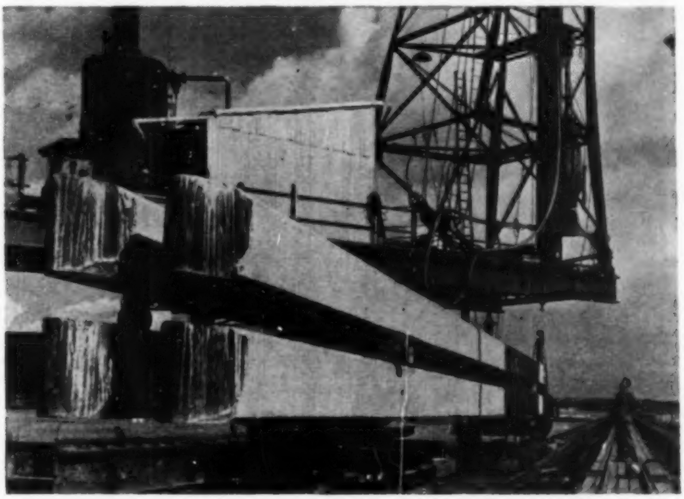

Fig. 3.—Rectangular Piles on Site Railway.

The Site Railway.

The site railway had a gauge of 3 ft. and comprised about 2 miles of track. It extended as a single track, on an embankment alongside the pipe-line across the low-lying ground, from various sidings in the casting-yard to the shore. From the shore to the terminus behind the pierhead trestle, the railway was a double track (*Fig.* 2) carried on steel beams supported on timber-pile trestles at 20 ft. centres. The tracks, between which there was a 6-ft. gangway, were at 15 ft. centres. The piles, of which there were 420, were 12 in. square and 70 ft. long, and were driven by a 2-tons drop-hammer in a cantilever frame with a leader for each row of piles.

Near the site of the jetty, a branch led to a pontoon from which aggregates and cement were loaded on to barges carrying mixing plants. A branch also led to a berth 100 ft. by 42 ft. where the cylindrical piles and precast frames were loaded on to barges, which were towed to the driving positions. There were two fixed gantries spanning the track and barge-berth, each carrying a 10-tons chain-block which enabled the concrete members to be picked up at two points and placed on the barge (*Fig.* 2). Another branch proceeded to the berths where the caissons were cast.

Materials were transported on the railway in 1½-cu. yd. side-tipping skips, of which there were twenty-four. Long concrete and steel piles, frames, and similar long pieces were transported on double-bogies. Four 5-tons and one 3-tons Diesel locomotives were used.

Concrete Plant.

Much of the concrete was supplied from a mixing plant operated by Messrs. Foster Wheeler, Ltd., about a mile from the new refinery and casting yard. At this plant, which was described in this journal for May, 1951, air-entrained concrete was produced and was delivered in 3-cu. yd. lorries either to the casting yard or to the seaward end of the railway embankment whence it was conveyed to the jetty in skips on the railway. During night shifts, and at other times when supplies from this plant were not available, concrete without an air-entraining chemical was mixed on the site. There was a stand-by mixer in the casting yard, and mixing plants at the end of the embankment and on the staging by the caisson berth.

The two floating mixing plants previously mentioned were each on a barge carrying fine aggregate, coarse aggregate of two sizes, and cement. At the forward end was a mixer and a crane for handling skips of concrete. The materials were measured in boxes. The smaller floating plant, seen on the left in *Fig.* 4, was used for the concrete placed in the joints between the precast members of the pierhead trestle and berthing islands. The larger plant, which had a 14/10 mixer, was used for concreting the decks of these structures, the concrete being deposited through a short length of chute.

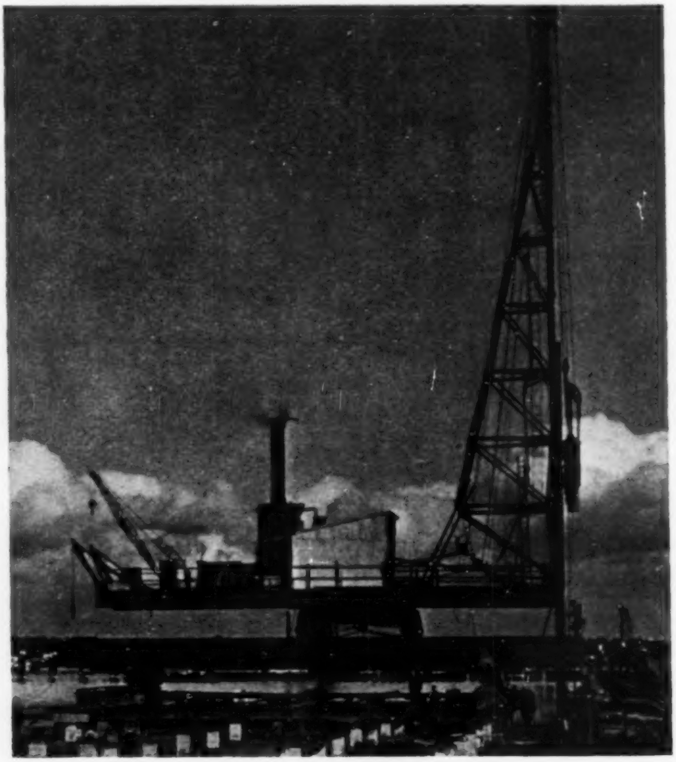

Fig. 4.—Driving Rectangular Piles.

Rectangular Piles.

The 16-in. by 14-in. rectangular piles, of which there are 637 in the approach, are from 55 ft. to 85 ft. long and weigh up

Fig. 5.—Driving Rectangular Piles with 4-tons Hammer.

to 9 tons each. Under each ordinary transverse frame there are three vertical piles and two piles inclined at a batter of 1 : 3, or in some cases four vertical piles. The frames are at 20 ft. centres and the distance between the outer piles is 47 ft.

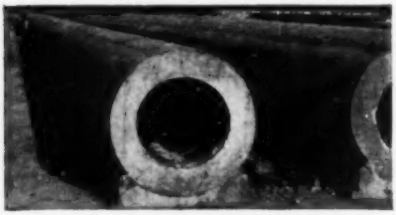

Fig. 6.—Cylindrical Piles on Concrete Mould-base.

Fig. 7.—Transporting a Cylindrical Pile.

The piles were brought from the casting yard to the site on the temporary railway (*Fig.* 3) parallel to the approach, and were lifted by a three-point suspension by the pile-driver acting as a crane.

The vertical piles were driven by a cantilevered frame (*Fig.* 4), the undercarriage of which comprised two transverse steel beams 14 ft. apart carried on two bogies running on longitudinal monorails 47 ft. apart. The frame was carried on steel cantilevers supported on and rotated about a four-wheel bogie on rails on the transverse beams. The longitudinal beams were supported on a lower tier of transverse beams bearing on piles already driven, the heads of which were cut off and protected by a steel cap. The length of the cantilever was such that piles were driven 20 ft. ahead of those supporting the undercarriage. The set to which the piles were driven was 12 in. for 120 blows from a 4-tons single-acting steam hammer (*Fig.* 5) falling 18 in. The points of the piles, which are driven to gravel overlying clay, are wedge-shape and are protected by steel plates. The inclined piles were driven from an ordinary pile-frame travelling on a grid laid on the vertical piles.

Cylindrical Piles.

There are 503 32-in. diameter precast hollow piles 75 ft. to 90 ft. long weighing up to 18 tons each in the trestles and berthing islands. The thickness of the

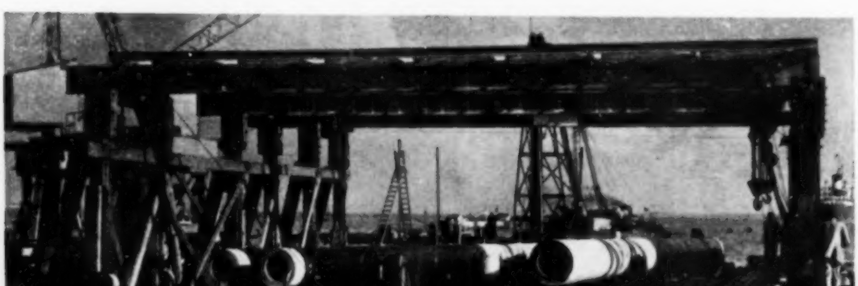

Fig. 8.—Berth for Loading Precast Members on to Barges.

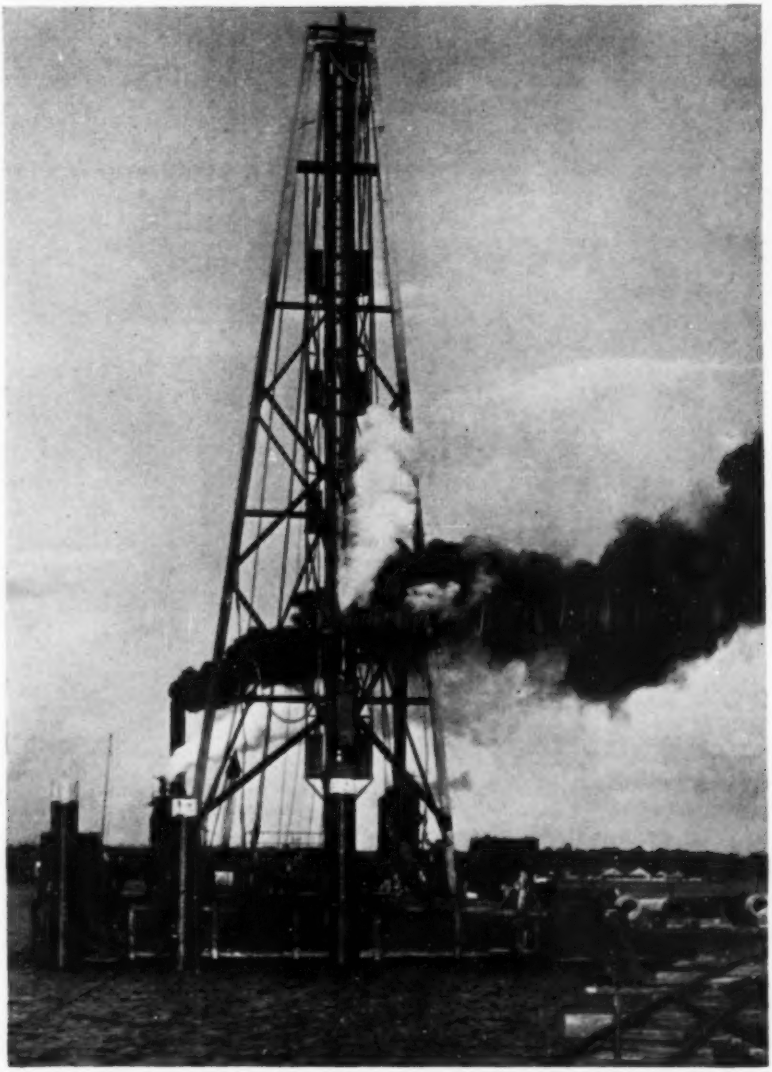

Fig. 9.—Driving Cylindrical Piles with 8½-tons Hammer.

wall of the pile is 5¾ in. The moulds were in five separate parts. The lowest part was a segmental concrete cradle, seen in *Fig.* 6, cast on the bed of the casting yard for the full length of the pile. The cradles, of which there were 38, formed the bottom quadrant. The two side quadrants were shaped by shutters in short lengths made of stout timber templates with steel plate attached. The upper quadrants were shaped by two steel shutters which hinged about the shutters below, and were brought into position when the concrete had been placed in the lower half of

the pile. The hinged shutters did not meet at the top of the pile, a gap being left for placing the concrete. The top face of the concrete in the gap was finished by hand. The shuttering for the cylindrical cavity was a collapsible core. The lower ends of the piles, which are flat, are protected by welded steel plates.

driven after the site had been dredged to the general level of about 41 ft. below datum, that is to 34 ft. below low-water at ordinary spring tides.

Two 90-ft. floating pile-drivers, which could be converted to cranes, were brought by sea from Holland. Each was mounted on a pontoon 60 ft. wide and

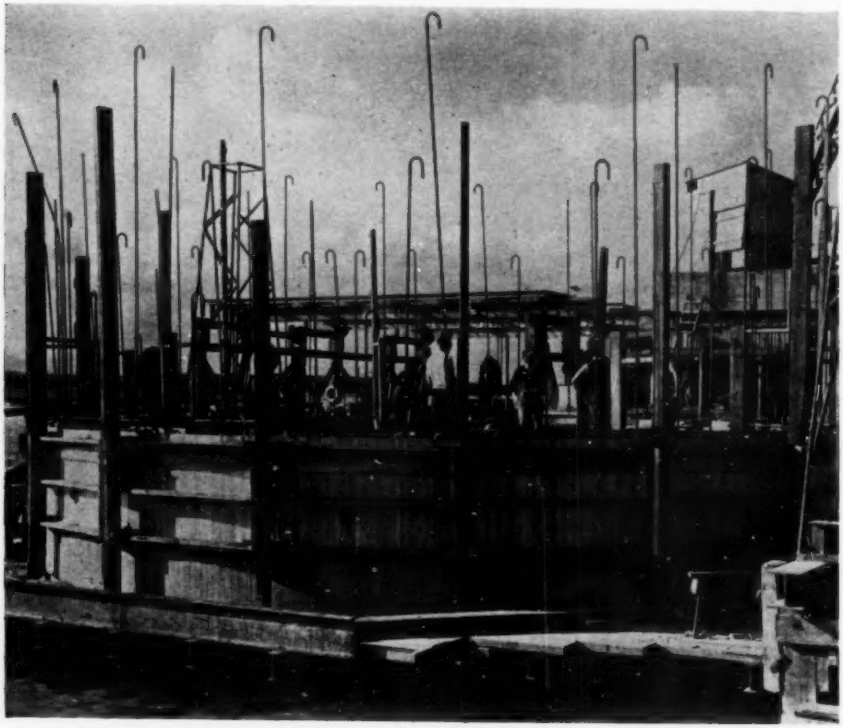

Fig. 10.—First Stage of Construction of Caisson.

The cage of reinforcement bars for a pile was fabricated in sections on a trestle immediately over the cradle on to which it was lowered by hand.

When the concrete had hardened, the four pieces of shuttering were removed and the piles remained on the cradle (*Fig.* 6) before being transported by the site railway (*Fig.* 7) to the berth where they were placed on a barge (*Fig.* 8) carrying four piles at a time to floating pile-drivers (*Fig.* 9). The piles were

70 ft. long. When at anchor, the drivers could operate over an area 100 ft. by 40 ft. The hammers were 8½-tons single-acting steam hammers. About fifteen piles were driven weekly by each pile-frame. The bearing strata are gravel and clay, into which the piles were driven to a set of 80 blows to 12 in. when the drop of the hammer was 2 ft. The heads of the piles were cut off at the level of the seating of the precast frames carried by the piles, but the reinforcement projected

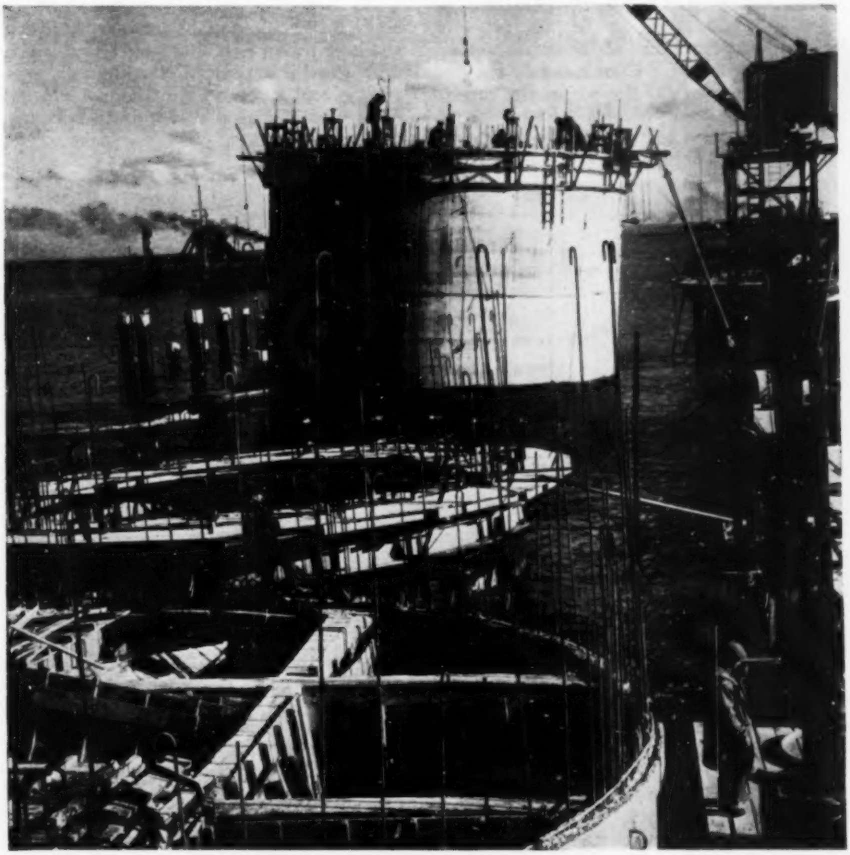

Fig. 11.—Foreground : Caisson on Platform (Jacks to the right). Middle : Caisson Afloat (lower part of wall under Construction). Background : Upper part under Construction.

3 ft. above this level so as to bond with cast-in-situ concrete forming the seating of the precast frames.

Caissons.

The two berths for casting caissons were immediately behind the pierhead trestle and each comprised two principal parts. In the first part a timber platform on steel beams (*Fig.* 10) was suspended by rods from four hydraulic jacks each of 120-tons capacity for the 40-ft. caissons and 70 tons for the 36-ft. caissons. The jacks were supported on a timber trestle. The bottom and 24 ft. of wall of a caisson were cast on the platform. After seven days the platform was lowered and at high-water the caisson was floated to the second part of the berth where ballast concrete was added to increase the draught from 9 ft. to 15 ft. The wall was then constructed up to a height of 45 ft., and more ballast added to give a draught of 29 ft. The wall was then completed to a total height of 66 ft., the draught being 33 ft., and the caisson was towed to and temporarily sunk at a convenient place for storage. Some stages in the

construction of the caissons are seen in *Figs.* 10 to 12.

The construction of a caisson took about five weeks. Continuously-moving forms were used for the walls, the rate of construction being from 4 in. to 6 in. per hour. Concrete for the walls constructed while on the platform was mixed at a plant at the casting berth and hoisted in wheelbarrows by a 15-cwt. mast-hoist. While the walls did not exceed 45 ft. high, concrete was placed by a 10-tons derrick crane having a 100-ft. jib and, when exceeding this height, by a 3-tons crane on a 30-ft. tower (*Fig.* 11).

At the permanent position of each caisson a hole was dredged to gravel about 57 ft. below datum. Immediately before sinking the caisson, silt was removed from the hole by an 8-in. compressed-air pump, and gravel deposited through an 8-in. tremie and screeded by divers to form a level bed. Water was pumped from a caisson in storage and the caisson was towed to the permanent position into which it was manœuvred by two craft just before high-water. The caisson was then sunk at the rate of about 1 ft. per minute by admitting water through two 10-in. valves. When grounded, the caisson was partly filled with sand. At this stage the wall projected about 3 ft. above high-water and was continued upwards by concrete cast in panel shutters.

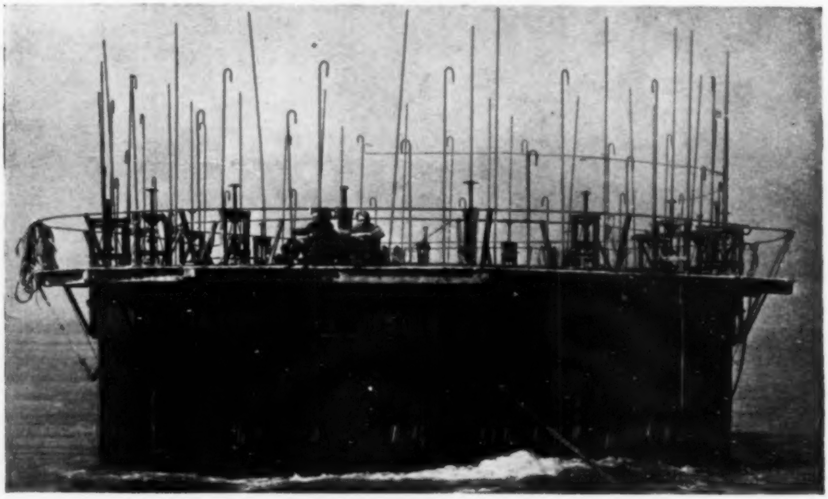

Fig. 12.—Caisson Being Towed, showing Moving Form.

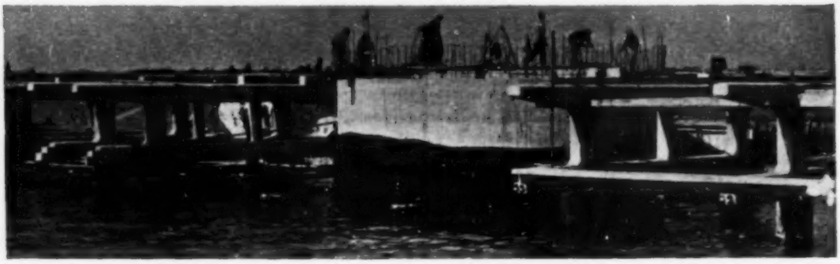

Fig. 13.—Part of Pierhead and a Caisson in Position.

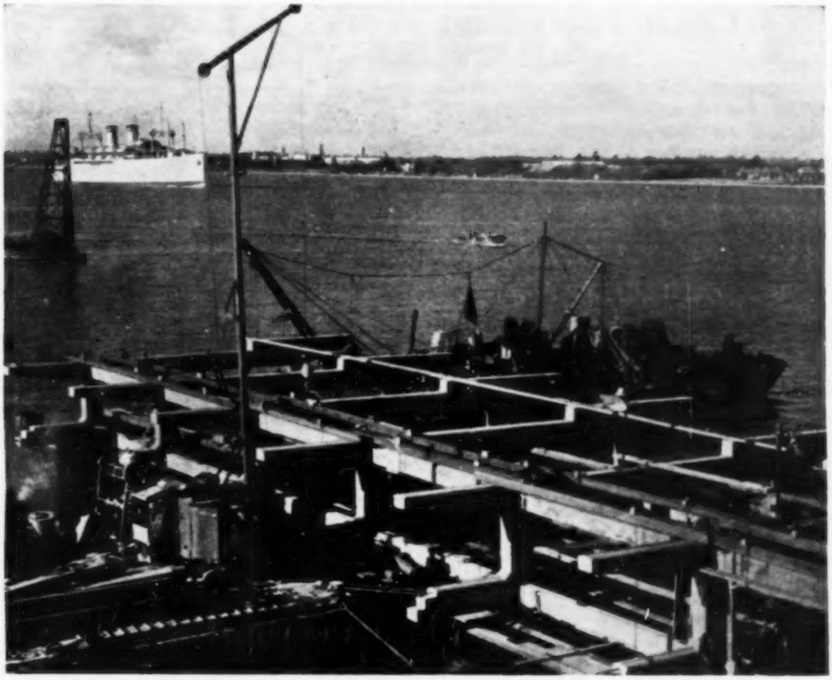

Fig. 14.—Part of Pierhead Trestle: Floating Mixer on left.

Precast Frames.

In the pierhead-trestle, berthing islands, and connecting trestles there are about 170 precast frames (*Figs.* 13 and 14), those in the pierhead-trestle weighing about 18 tons each and in the berthing islands 10 tons each. There are more than 1000 precast struts and gangways. Each frame was cast on a concrete base of the same shape as the elevation of the frame and projecting above the casting bed. Twenty bases were used. After the reinforcement had been placed, wooden side shutters were placed against the edges of the base which acted as a template for the mould. When a frame was sufficiently matured, it was taken on the site-railway to the same loading berth as the cylindrical piles, and thence by barge to the erection position. The precast struts were dealt with in the same manner.

The frames and struts on one side of the structure were lifted into position by a 30-tons floating crane, and the struts on the other side by a 3-tons floating crane. Cylindrical steel caps (*Fig.* 13) attached to the top of the cylindrical piles supported the frame temporarily and also supported the staging and working platform and served as shuttering for the concrete cast in situ to support permanently the frame.

The contractors for the civil engineering work are Messrs. Christiani & Nielsen, Ltd., who also designed the civil engineering structures in collaboration with the Standard Oil Development Corporation with the exception of the approach which was designed by the Corporation. Construction commenced in January, 1950, and the first berths were completed in August, 1951; it is expected that the remainder of the work will be complete in the autumn of this year. The total cost of the civil engineering work is about £1,600,000. At the peak period of construction about 1250 workmen, 15 general foremen, 30 trades foremen, and 25 clerks, storekeepers, etc., were employed in addition to 25 civil and mechanical engineers.

Coast Protection Works at Clacton.

COAST-PROTECTION works comprising concrete groynes and a sea-wall at Clacton-on-Sea are described by the Borough Engineer and Surveyor, Mr. W. Aiston, in a recent number of the Journal of the Institution of Municipal Engineers, from which the following is abstracted.

The Main Groynes.

Owing to the deterioration of existing timber groynes, all new groynes are of concrete and are 310 ft. long (*Fig.* 1). At the landward end are a ramp and steps, and a sluice with timber planks by which the drift of sand and shingle can be partly controlled. Seaward from the sluice the groyne is of plain concrete and is of constant shape but decreasing size. The sides are vertical, and buttresses and steps are provided to break scour action. The toe-piles are 12-in. by 6-in. or 7-in. reinforced concrete sheet-piles with bird-mouth joints and 10 ft. to 15 ft. long. The groyne is divided into bays each 11 ft. 6 in. long keyed together by a tongue-and-groove joint for the full height. Dowel-bars 5 ft. long and 1¼ in. diameter are provided at the bottom across each joint. Every third joint is an expansion joint having a ½-in. gap filled with bituminised cork.

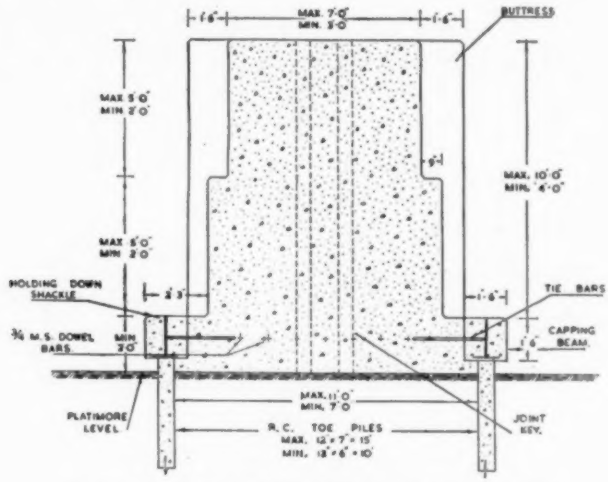

Fig. 1.—Typical Cross Section of Main Groyne.

The concrete in the core of the wall is 1 : 2 : 4 with gravel aggregate graded from 1½ in. to ⅜ in. The exposed faces are finished with 4-in. of 1 : 1½ : 3 concrete, the aggregate in which was, in the early parts of the work, ⅜-in. granite chippings. Tests indicated that the resistance of gravel aggregate to abrasion from beach material was as good as that of the granite aggregate. The face of granite concrete was therefore omitted in later parts of the work, and gravel graded from ⅜ in. to ¼ in. used.

Each groyne was constructed within a cofferdam extending above high-tide level. Excavation was carried down to a firm clay, called platimore, and the concrete sheet-piles were driven. The six longitudinal reinforcement bars in each pile projected 5 in. above the head, and the ends were bent outwards to key into the 18-in. square capping beam. The shuttering for the concrete in the groyne was steel-faced timber and, where the 4-in. cover of fine granite concrete was provided, 18-in. by 12-in. sliding steel plates were used to separate the two concretes. The concrete was delivered to the shutters in 1-cu. yd. skips and was spread over the area of one bay.

The greatest amount of concrete to be

placed in one bay was about 50 cu. yd. and the average time taken to place this amount was about 7½ hours when a 4-in. granite-concrete cover was provided and 5½ hours when using concrete of one quality only. In the part of the groyne at the steps about 170 cu. yd. of concrete were placed in about 22 hours. About 1250 cu. yd. of concrete were placed in the first groyne in addition to the concrete in the piles.

Intermediate Groynes.

Intermediate groynes, which were constructed in 1942–43, are at about 330-ft. intervals and extend 100 ft. seaward from the wall. They comprise 12-in. square reinforced concrete grooved piles from 15 ft. to 28 ft. long at 6-ft. centres. Reinforced concrete slabs 5 ft. 4 in. long and 11 in. by 5 in. in cross section fit in grooves in the piles.

The Sea-wall.

The new sea-wall has three horizontal 10-in. steps in addition to a batter on the seaward face (*Fig.* 2). To protect the foundation against scour, 13-in. by 7-in. V-jointed reinforced concrete sheet-piles 10 ft. 6 in. long are driven to below mean low-water level in front of the wall. Above the piles is an 18-in. square reinforced concrete capping-beam. Each bay is 15 ft. long, adjoining bays being keyed together by a tongue-and-groove joint and seven 1¼-in. mild steel dowel-bars near the bottom. Allowance is made for expansion at alternate joints, in which a ½-in. slab of bituminised cork is inserted. A water-seal is provided by a copper strip across the joint.

The length of the wall is 660 ft. in a straight length which for constructional purposes was divided into five parts each about 130 ft. long and each built within a steel sheet-pile cofferdam. At no time did work proceed on more than two parts. After excavating, the reinforced concrete sheet-piles were driven and the capping-beam constructed. The shuttering for the wall was a steel travelling shutter (*Fig.* 3) suspended inside a gantry by three jacks at both ends. The gantry moved on a track one rail of which was on the capping-beam and the other on the ground behind the wall. Striking, moving, and re-setting were completed in a few hours. The length of the shutter was 15 ft. 3 in., and overlapped the completed part of the wall by 3 in.

The wall is of 1 : 2 : 4 concrete. An immersion vibrator was used for compacting the concrete at the stepped sections of the wall, the remainder being

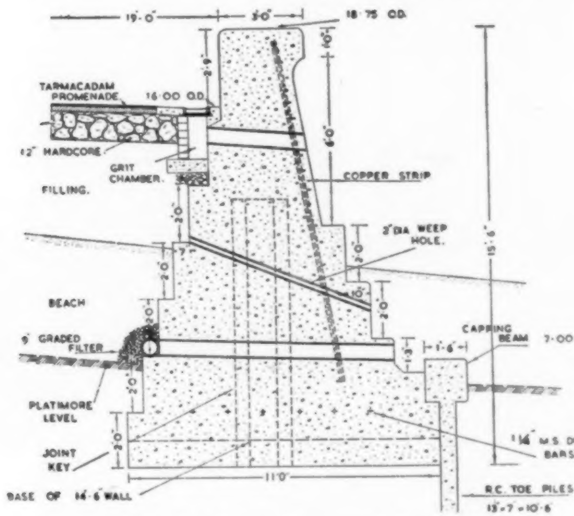

Fig. 2.—Cross Section of Sea Wall.

consolidated by treading and hand punning. In each 15-ft. bay there are about 50 cu. yd. of concrete which were placed in six hours. The cost of the sea-wall was about £35 per foot.

As it was possible to construct only a short length of sea-wall at present, temporary protection of the remainder of the foreshore is provided by a wall of 13-in. by 7-in. reinforced concrete sheet-piles 21 ft. long driven 4 ft. below mean low-water level and stiffened by timber walings and struts bearing against timber piles. When the sea wall is extended the upper part of the piles will be cut off and a reinforced concrete capping-beam cast on top. Two pile-frames worked continuously on this temporary wall, and each frame drove up to thirteen piles a day through about 2 ft. of beach material and 12 ft. of clay containing small clay stones, which caused difficulties in driving. Often by hard driving the piles displaced the stones, but in some cases the stones caused the piles to twist slightly or become inclined. The distorted piles have not been withdrawn because of the difficulty of so doing and redriving them plumb, and the only detriment to the wall is its appearance. When the construction of the sea-wall proceeds, the alignment will be trued by the capping-beam. The cost of this wall, including drains, timber piles and struts, was about £18 per ft.

The contractors were Messrs. G. Percy Trentham, Ltd.

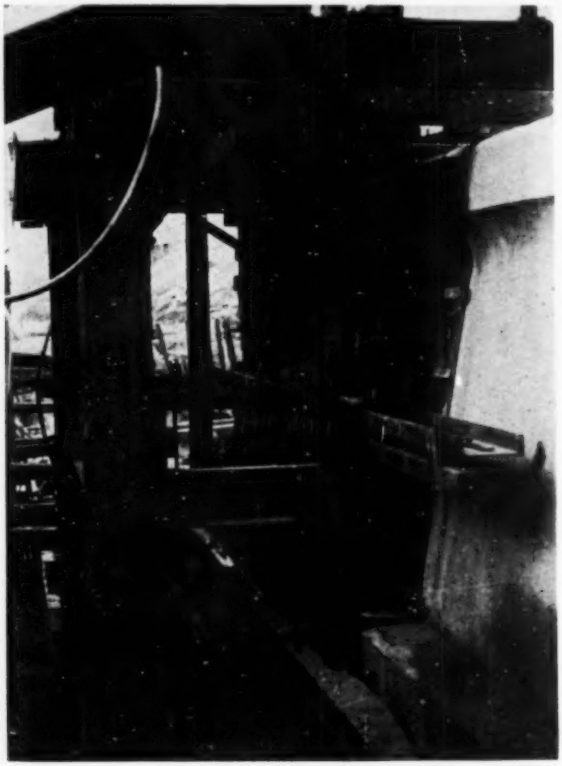

Fig. 3.—Travelling Shutter for Sea Wall.

Effect on Concrete of Chemicals used in Consolidating Soils.

We have submitted to Imperial Chemical Industries, Ltd., an inquiry on the effect on concrete of the chemicals commonly used in consolidating soils, and as this is a matter of general interest we give their reply below.

"The chemicals most frequently used in two-fluid injection processes for chemically consolidating loose ground or porous strata are solutions of sodium silicate and calcium chloride. Sodium carbonate is sometimes added to the sodium silicate solution. There are also single-fluid processes based on sodium silicate solutions containing chemicals such as sodium bicarbonate, sodium aluminate, and borax. In the two-fluid process the end product is usually considered to be a complex mixture of gels of hydrated silica and sodium silicate with a small amount of calcium silicate, whilst in the single-fluid process gels of hydrated silica predominate. The formation of any calcium silicate by interaction between calcium chloride and sodium silicate will be accompanied by the formation of an equivalent quantity of sodium chloride.

"In our opinion the end products of these reactions can have no effect either on precast concrete or on concrete placed in position in contact with them. These end-products are inert and neutral, and are not able to change in course of time into anything that could damage concrete. Neither do we think that any sodium silicate or calcium chloride that may have been added in excess, or have failed to react, will have any significant effect on precast concrete, since neither chemical is harmful to concrete ; sodium silicate in fact has a beneficial hardening effect. We should not be so confident if sulphates were used in any of these processes (for example aluminium sulphate has, we believe, been used on occasion in the single-fluid process) since Portland cement

concrete in contact with soluble sulphates is liable to suffer disintegration. With regard to the effect, if any, that sodium silicate or calcium chloride could have on concrete placed in position in contact with them, as both chemicals accelerate the setting and hardening processes of Portland cement it would no doubt be possible for a very thin layer of the concrete in contact with the consolidated ground to set and harden somewhat more quickly than the rest of it. Even if this did happen we do not think it would have any significant effect on the concrete as a whole, for it is likely that the amounts of sodium silicate or calcium chloride liable to be present after a consolidation process are so small that their effects can be ignored. There is in our opinion no danger of concrete in contact with ground containing traces of sodium silicate, gels of sodium silicate or silica, calcium chloride, sodium chloride, sodium carbonate, or sodium bicarbonate suffering any slow attack over periods of years.

"Most of the information given in this letter is clearly a matter of opinion and, while our opinion is given in good faith, we cannot accept responsibility should time prove it to be wrong."

Composite Trussed Beams.

Fig. 1.

To reduce the weight of precast concrete beams of long span, Dr. Ernst Deutsch describes in "Beton-und Stahlbetonbau" for March, 1951, a type of composite concrete and steel trussed beam used for the roof of a factory (*Fig.* 1). The compression flange is a rectangular precast concrete beam 13¾ in. deep and 12 in. wide reinforced with four $\frac{9}{16}$-in. bars in the top and bottom. The tensile flange and vertical members are welded steel channels. The trussed beams span about 33 ft. and are designed for a total load of 1344 lb. per ft. The tensile stress in the bottom tie is 19,900 lb. per square inch. The fabrication of the trussed beams is simple. The channels are welded together first and are then placed upside down in the mould for the concrete beam; plates welded to the ends of the channels enable the steel frame to stand in the mould. The trussed beams are at 16 ft. centres and support precast slabs.

Loading tests were made on two prototypes of the trussed beams to determine the factor of safety in bending and shear, and to prove the stiffness of the composite member and the effectiveness of the bond between the steel channels and the concrete beam. The results of the tests were satisfactory, although under unsymmetrical loading it was possible to cause local overstressing of the concrete.

This type of composite trussed beam effects a great saving of steel compared with a steel truss, and a great saving of weight compared with a concrete beam.

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(*Continued on page* liv.)

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(*Continued from page* liii.)

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